

Technical University of Denmark



Assesment of Severely ASR Damaged Bridges: From Diagnosis to Structural Effects

Barbosa, Ricardo Antonio; Hansen, Søren Gustenhoff; Hansen, Kurt Kielsgaard; Hoang, Linh Cao; Grelk, Bent; Maag, Iben

Published in:

Proceedings of the 15th international conference on alkali-aggregate reaction in concrete (15th ICAAR)

Publication date:

2016

Document Version

Peer reviewed version

[Link back to DTU Orbit](#)

Citation (APA):

Barbosa, R. A., Hansen, S. G., Hansen, K. K., Hoang, L. C., Grelk, B., & Maag, I. (2016). Assesment of Severely ASR Damaged Bridges: From Diagnosis to Structural Effects. In Proceedings of the 15th international conference on alkali-aggregate reaction in concrete (15th ICAAR)

DTU Library

Technical Information Center of Denmark

General rights

Copyright and moral rights for the publications made accessible in the public portal are retained by the authors and/or other copyright owners and it is a condition of accessing publications that users recognise and abide by the legal requirements associated with these rights.

- Users may download and print one copy of any publication from the public portal for the purpose of private study or research.
- You may not further distribute the material or use it for any profit-making activity or commercial gain
- You may freely distribute the URL identifying the publication in the public portal

If you believe that this document breaches copyright please contact us providing details, and we will remove access to the work immediately and investigate your claim.

Assessment of Severely ASR Damaged Bridges: From Diagnosis to Structural Effects

Ricardo Antonio Barbosa¹, Søren Gustenhoff Hansen², Kurt Kielsgaard Hansen¹, Linh Cao Hoang¹,
Bent Grelk^{1,3}, Iben Maag⁴

¹Technical University of Denmark, Kongens Lyngby, DENMARK

²University of Southern Denmark, Odense, DENMARK

³Grelk Consult, Copenhagen, DENMARK

⁴The Danish Road Directorate, Copenhagen, DENMARK

Abstract

During the last decade an increasing number of bridges are becoming severely deteriorated due to alkali-silica reaction (ASR) in Denmark. Some bridges have already been demolished due to lack of knowledge on the residual load carrying capacity.

The deterioration of concrete by ASR has been widely documented. The majority of the experimental data are based on relatively small scale laboratory specimens accelerated by various exposure conditions. Research on assessment and influence of severely ASR deterioration on the material properties and residual load carrying capacity of real-life structures is unfortunately limited.

This paper presents an overview and discussion of the Danish experiences with assessment of the residual load carrying capacity of severely non-shear reinforced ASR damaged bridges. The discussion is supported by experimental data acquired from large scale in-situ tests of three severely ASR deteriorated bridges. The influence of ASR cracking on the mechanical properties of concrete and the pre-stress effect on the reinforcement due to ASR expansion are discussed.

Keywords: ASR, bridges, pre-stress, compressive strength and shear capacity

1 INTRODUCTION

Alkali-silica reaction (ASR) is a well-known deterioration mechanism in Denmark. In Denmark most of the aggregate sources contain reactive silica components both in the fine and in the coarse aggregate fraction. Porous opaline flint and porous chalcedonic flint were found as the main reactive aggregate types in the country [1]. Both reactive aggregate types are characterized as fast reactive, which can cause deleterious ASR cracks within a few years in the structures [2].

Systematic research on ASR in Denmark started already back in 1951 based on the report of Poul Nerenst [3]. The early research on ASR was performed by the Danish Committee on Alkali Reaction in Concrete. An important contribution of this research programme was the classification of flint types. Another significant contribution was the statement: if the porous flint content in the sand is less than 2 vol. %, the sand is less liable to cause deleterious expansions and crack formations in the concrete [1]. The early research also resulted in a systematic approach to the assessment of deteriorated concrete structures [4]. From 1970 research on ASR in Denmark consisted of both laboratory and real-life structures. Among others the external supply of alkali e.g. from de-icing salts was investigated [5]. This research period led to the development of a mortar bar expansion test in saturated sodium chloride solution, test method TI-B51. The mortar bar test measures the potential for aggregates to expand [6].

In 1961 a comprehensive research on ASR already existed in Denmark and preventive measure concerning the amount of reactive porous flint was available for the concrete industry [1]. However, there was no official regulation or requirements concerning the amount of reactive compounds in aggregates and in alkali content in cement until 1987. In 1987 a Code of Practice was published [7]. This Code of Practice specifies a concrete specification for all public and governmental constructions and is used to prevent deleterious ASR. The Code of Practice provides requirements for the maximum alkali content of the concrete and the acceptance criteria of the reactivity of aggregates with correlation to the expected environmental exposure of the structure. By fulfilment of the requirements

* Correspondence to: ranba@byg.dtu.dk

in the Code of Practice it is expected that the incidence of deleterious ASR in Denmark will be limited in the future.

However, since the Code of Practice first was adopted and made mandatory for public and governmental constructions in 1987, today Denmark is in a situation where a large percentage of the Danish bridges and tunnels are constructed with concrete containing a critical amount of reactive aggregates. The reactive aggregates are mostly found in the sand fraction as porous opaline and chalcedonic flint. The Danish Road Directorate has estimated that approximately 600 bridges have the potential to develop severe ASR, which corresponds to approximately 15 percentages of the Danish Road Directorate owned bridges. A large amount of the bridges are constructed during the rapid development of the Danish infrastructure in the 60s and 70s. In addition to the 600 of the Danish Road Directorate owned bridges there may be an unknown but not less significant amount of municipal bridges constructed of concrete containing a critical amount of reactive aggregates.

Today, some bridges in Denmark are suffering from severe ASR with comprehensive ASR induced crack formation. To the authors' knowledge four bridges in Denmark had already been demolished due to ASR deterioration and uncertainties regarding the residual shear capacity and safety requirements of these bridges. However, the decision to demolish these bridges has been based on a subjective evaluation built on visual inspections and petrographic analysis of drilled cores from the bridges.

Realistic full-scale tests of real-life ASR deteriorated structures are limited. Up to 2015, most of the tests regarding the influence of ASR deterioration on the shear capacity of reinforced concrete are based on small-scale laboratory prepared and conditioned elements [8,9,10,11]. Test results from different authors concerning the shear capacity of ASR deteriorated reinforced concrete are generally contradictory. This may be due to differences in the degree of damage, reinforcement configuration, test setup, type of reactive aggregate and the conditioning of the tested elements. These differences result in significant challenges in the interpretation and comparison of test results.

Therefore, the Technical University of Denmark, University of Southern Denmark and The Danish Road Directorate took the initiative to start a project where to date three real-life severely ASR deteriorated bridges were investigated. The purpose is to provide more experimental test results of real-life severely ASR deteriorated bridges and to obtain a profounder understanding of the parameters influencing the residual shear capacity of these bridges. Three of the major civil engineering consultant companies in Denmark, Rambøll, COWI and NIRAS participate in the project.

This paper presents an overview and discussion of the Danish research from the diagnosis of ASR to the structural effect of real-life ASR deteriorated bridges. The paper will among other show preliminary results of the influence of crack orientation on the compressive strength of concrete and the shear capacity of severely ASR damaged beams cut from a real-life ASR damaged bridge.

2 THE EXAMINED REAL-LIFE ASR DETERIORATED BRIDGES

From 2012 to date three severely ASR deteriorated bridge decks were examined and full-scale tested. The bridge decks in all the investigated bridges were only provided with two layers of horizontal reinforcement but not provided with vertical reinforcement. It is well-known that crack formation due to ASR strongly depends on the boundary conditions and on reinforcement configuration of the structures. Since all the bridge decks were not provided with vertical reinforcement, the cracks were mainly orientated parallel to the horizontal reinforcement inside the decks. The ASR expansion in the vertical direction is not constrained by any reinforcement. Figure 1 shows the typical crack formation in the bridge decks. The fluorescent impregnated concrete cores in Figure 1 are drilled both vertically and horizontally from the bridge deck in bridge no. 3.

The background of the examined bridges and the tests performed on each bridge are as described:

Bridge no.1 was constructed in 1976 and consists of three spans. Bridge no. 1 is a beam and slab bridge. The middle span is located above a highway and the side spans are located above hillsides at the bridge ends. The bridge was constructed in reinforced concrete, where the non-shear reinforced bridge deck is cast together with two longitudinal prestressed main beams. The total bridge length is 54.5 m and the width is 10.0 m. A visual inspection of the bridge in 2012 showed comprehensive fine wet longitudinal ASR induced cracks and white precipitations at the cantilever part and at the prestressed main beams. Figure 2 shows the ASR induced cracks in the northern cantilever part and prestressed main beams. Four in-situ full-scale shear tests were performed on the northern cantilever part of the bridge deck. Results from these full-scale tests are presented in [12] and a profounder analysis and measurements are presented in [13]. Based on the in-situ full scale shear tests a demolition of the bridge was avoided with significant economic savings for the Danish Road

Directorate and for the society in general. Besides of the in-situ shear tests, 12 beams from the cantilever part were cut and send to the laboratory for core drilling and for petrographic analysis. The drilled cores were used to measure the influence of ASR crack orientation on the compressive strength and on the splitting tensile strength of the concrete. In 2012, the widths of the cracks were in average measured to 0.1 mm and the maximum widths of the cracks were 0.25 mm. The widths of the cracks were measured on 12 beams.

Bridge no. 2, was also constructed in 1976 and consisted of three spans. Bridge no. 2 was a slab bridge. The middle span was 14.0 m and the side spans were 8.25 m. The middle span was located above a highway. The entire bridge was constructed in reinforced concrete. The total bridge length was 30.5 m and the width was 9.1 m. The height of the bridge deck was 0.7 m. The entire bridge deck was cracked due to ASR. The bridge was demolished in 2010 due to uncertainties regarding the residual shear capacity of the bridge deck. In connection with the demolition of the bridge an in-situ full-scale test was conducted on a part of the bridge. No reduction of the shear capacity due to ASR was reported [14]. However, four 7.65 m long beams were sawed and stored until 2013. In 2013, the widths of the cracks in the beams were in average 0.1 mm and the maximum widths of the cracks were locally measured to 4 mm at the end of the beams. The four beams were tested in a three point bending test setup in the laboratory. Cores were drilled from the beams and strain measurements on the reinforcement were performed after locally removing the concrete cover for the beams.

Bridge no. 3, was constructed in 1966-1967. Bridge no. 3 is a slab bridge. The pile supported slab bridge consists of a southern and a northern bridge. The total length of the northern bridge is 312 m and the total length of the southern bridge is 120 m. The width of the bridge is 10.0 m. The height of the deck is 0.3 m. Figure 3 shows an overview from beneath of the bridge deck. Both the northern and the southern bridge decks are completely cracked due to ASR. Already 10 years after the inauguration of the bridge major signs of ASR induced cracks were observed underneath the bridge deck. Figure 4 shows cracks observed from beneath of the bridge deck in 2014. In 2014 six trapezoidal slabs were cut from the northern bridge deck. The slabs were cut only from the northern bridge. Each of the six trapezoidal slabs was cut into six beams with different lengths. In 2014, the widths of the cracks in the beams were in average 0.1 mm and the maximum widths of the cracks were measured to 1.5 mm. Figure 5 shows the trapezoidal slabs after the cutting into beams at the Technical University of Denmark. The beams were marked 3.1, 3.2, 3.3 and A, B, C, D, respectively. Full-scale bending tests of beams marked 3.1, 3.2 and 3.3 were performed in the laboratory. The test setup and the shear span to effective depth ratio were varied during the test of the 18 beams cut from six slabs. The reinforcement configuration was the same in all 18 beams. Strain measurements on the reinforcement by strain gages were performed on beams marked A, B, C and D from all six slabs after locally removing the concrete cover. Concrete cores were drilled from beams marked A, B, C and D with different orientations from all the six trapezoidal slabs.

A detailed visual inspection has been conducted on all beams and elements cut from the three ASR damaged bridge decks. Moreover several examinations of thin sections have been conducted. From these examinations and inspections the authors can in general neglect the possibilities for other contributing factors to cause cracking. The contributing factors could be fatigue of concrete slab by repeated active loading by traffic, freezing and thawing and steel corrosion due to chloride intrusion. However, locally in some of the beams cut from bridge no. 3 signs of steel corrosion due to chloride intrusion have been observed, see Figure 9.

3 EXPERIMENTAL

3.1 Uniaxial compressive strength and crack orientation

The majority of literatures reported considering the compressive strength of ASR deteriorated concrete are related to the correlation between compressive strength reduction and ASR induced expansion. In these studies the concrete cores are prepared and conditioned in the laboratory without taking into account the restraintment of the ASR expansion by the reinforcement. The crack patterns of these laboratory conditioned concrete cores are generally irregular leading to so-called map-cracks. Furthermore, it is recognized that there are significant variations in the crack patterns between laboratory conditioned cylinders and real-life drilled cores. This is due to discontinuity in the crack formation and variation in environmental conditions.

When assessing the residual load carrying capacity of real-life structures the actual compressive strength of the drilled concrete cores is essential in the evaluation.

As mentioned in the previous chapter, in all of the investigated real-life bridge decks the crack orientation is very regular. The cracks are generally orientated parallel to the main horizontal reinforcement in the bridge decks, since the decks are not provided with vertical reinforcement. In

order to clarify the influence of the crack orientation on the uniaxial compressive strength of ASR cracked concrete, concrete cores were drilled from beams from all three investigated bridge decks. The cores were drilled both vertically and horizontally.

A diamond drill cooled with water circulation was used for the drilling. The large majority of the drilled concrete cores from the three bridge decks were surprisingly cohesive after drilling. Despite of the comprehensive formation of ASR cracks it may be assumed that the reaction product, the alkali-silica gel, probably act as a strong glue binding the crack walls together.

After drilling the ends of the concrete cores were cut and squared by polishing. Generally, concrete near the top and bottom of the drilled cores were cut off, since the deterioration degree can be different above and under the reinforcement bars. After drilling and cutting the concrete cores were of 100 mm diameter with a length/diameter ratio of 1.5 to 2.0. To avoid desiccation cling-film was wrapped around the concrete cores and afterwards sealed in plastic bags just after cutting and polishing. The concrete cores were packed until the uniaxial compressive strength tests were performed.

The compressive strength tests were performed as deformation controlled with a loading rate of 0.5 mm/min. This differs from the European standard [15] where compressive strength tests are performed as load controlled testing. After the compressive strength test the measured concrete core strength was converted to a concrete strength corresponding to a 150 x 300 mm cylinder according to guidelines by the Danish Road Directorate [16].

3.2 Strain gage measurement of ASR induced pre-stress in reinforcement

In the literature where the increases in shear strength are reported it is commonly stated that the decrease in compressive strength is compensated by the pre-stress induced in the reinforcement as a result of the ASR expansion. In order to verify whether the ASR induced expansion has caused a pre-stress effect in the longitudinal reinforcement, tension strain measurements were performed on beams from bridge no. 2 and no. 3. The measurements were performed after the elements are cut into beams.

Before bonding of the strain gage to the surface of the reinforcement bars the concrete cover was locally removed. The free part of the reinforcement bar was grinded and the surface was abraded to remove any loosely bonded adherents in order to develop a surface texture suitable for bonding. After preparation of the surface the strain gage was locally glued onto the reinforcement bar. A wire was soldered to the copper terminal on the strain gage and connected to a data logger. The preparation of the surface and bonding of the strain gage were performed according to [17]. The reinforcement bar was cut about 15 cm from the strain gage in the same time as measurements by the data logger were performed. The data logger measures the contraction of the reinforcement bars.

3.3 Shear capacity of real-life ASR damaged beams

From 2012 to date 22 ASR damaged beams were shear tested in the laboratory. Four beams were cut from bridge no. 2 and 18 beams were cut from bridge no. 3 for laboratory tests. In addition to the 22 beams 4 in-situ full-scale shear tests are performed in the cantilever part of bridge no. 1 as described above.

In this paper only shear tests of three beams from bridge no. 3 are presented. The three beams were cut from one trapezoidal slab. The beams were loaded until failure in a symmetric three point bending test setup. The shear span to effective depth ratio, a/d , varied from 2.56 to 3.92. The load was applied by a deformation controlled actuator with a loading rate of 0.5 mm/min. Deflections were measured at 8 points along the beam. Moreover, the Digital Image Correlation (DIC) system ARAMIS was used to monitor the load induced deformations and crack propagations during testing.

4 RESULTS AND DISCUSSION

In this chapter only a few results obtained during the project are shown and discussed. Generally, the results are representative for the three investigated reinforced bridge decks.

4.1 Uniaxial compressive strength and crack orientation

Figure 6 shows the influence of crack orientation on the converted compressive concrete strength of concrete cores drilled from bridge no. 3. The concrete cores are all drilled from the same trapezoidal slab in the same area. The converted compressive concrete strengths are represented graphically as a boxplot. Figure 6 shows clearly that the strength of the concrete cores with ASR cracks primarily orientated parallel (horizontally drilled) to the load direction are considerably higher than the strength of concrete cores with ASR cracks perpendicular (vertically drilled) to the load

direction. These results are supported by test results from the two other tested bridge decks. However, the amount of ASR cracks and the compressive strengths can be different between the bridge decks and areas within the same bridge deck.

These results are also supported by results reported by [18], which to the authors' knowledge is one of the very limited studies focusing on the influence of elastic restraintment on ASR crack formation and compressive strength of laboratory accelerated and conditioned specimens.

The modulus of elasticity, Young's modulus, for the ASR deteriorated concrete cores also depends on the crack orientation within the tested concrete cores. Figure 7 shows the stress-strain relationship for concrete cores with ASR cracks parallel and perpendicular to the load direction. The Figure shows that the modulus of elasticity of the concrete core with ASR cracks parallel to the load direction is 26.1 GPa and of the concrete core with ASR cracks perpendicular to the load direction is 2.5 GPa. The reduction in modulus of elasticity is significant for concrete cores with ASR cracks perpendicular to the load direction, which can be attributed to the compaction of the ASR cracks. To the authors' experience the considerable reduction in elastic modulus of concrete cores cannot directly be related to the possible reduction in the bending stiffness of ASR deteriorated reinforced concrete beams. The difference between the measured modulus of elasticity of ASR cracked cores and the bending stiffness of ASR cracked reinforced concrete beams may be attributed to restraint conditions of concrete in the reinforced beams and the ASR induced pre-stress in the reinforcement.

4.2 Strain gage measurement of ASR induced pre-stress in reinforcement

Figure 8 shows 6 strain gage measurements attained from beams cut from two different trapezoidal slabs from bridge no. 3. The Figure shows a rapid increment from zero strain when the reinforcement bar was cut, where after increases in the slope are seen and then a stabilization of the curves. This steep increasing part of the curves corresponds to the cutting of the reinforcement bars and thereby the contraction of them. The stabilized parts of the curves correspond to the measured tensile strain, i.e. pre-stress action, in the reinforcement bars. Figure 8 shows that the measured strains in the reinforcement bars are similar on approximately 1.0 ‰ in beams from both trapezoidal slabs. The value of 0.72 ‰ is lower than the other measurements which is explained by a much lower anchorage length (0.3 m) of the reinforcement bar compared to the anchorage length of the other reinforcement bars (>1.0 m).

Figure 9 shows the visual condition and crack formation on beams cut from the two trapezoidal slabs. The Figure shows that the beams cut from slab no. 2 contain remarkable fewer ASR cracks than beams cut from slab no. 1. The compressive strength of cores drilled from the two slabs is also different. In average the compressive strength is 15 MPa lower for slab no. 1 compared to slab no. 2 for both crack orientations. These results question how the pre-stress in the reinforcement develops as a function of ASR expansion and crack formation. Apparently, the above mentioned results show that the ASR induced pre-stress in the reinforcement may be achieved just as the first ASR cracks and expansion develops in the concrete. The ASR induced pre-stress action in the reinforcement may rapidly achieve a constant level even if further expansion and crack formation occur in the concrete.

The Technical University of Denmark and the Southern University of Denmark are currently performing tests and strain gage measurements on 16 non-shear reinforced slabs with dimension of 1.5 x 1.5 x 0.25 m (l x w x h). The slabs are cast in laboratory. Some of the slabs are conditioned in a climate chamber and some will be placed outdoor and exposed to a natural real-life environment, since it is not clear if the ASR induced pre-stress action in laboratory conditioned specimens develops to the same rate as outdoor. It is expected that the ASR expansion will be slower in a real-life environment. The research is performed in order to investigate how the pre-stress in the reinforcement develops as a function of ASR expansion and crack formation, and the relation between possible decrease in compressive and tensile strength.

4.3 Shear capacity of real-life ASR damaged beams

The shear test results for the three beams 3.1, 3.2 and 3.3 are shown in Figure 10. The Figure shows a linearly decrease of the shear capacity as function of the shear span to effective depth ratio, a/d . The high shear capacity at a small shear span to effective depth ratio may be explained by the effect of arching action.

The shear test results were compared with calculations based on the shear model in Eurocode 2 [19,20]. The use of this model is based on a practical point of view since the shear model in Eurocode 2 is the basis of shear verification in most European countries.

Table 1 shows the calculated shear capacities according to Eurocode 2. In the calculations the

average measured strain at the longitudinal reinforcement bars of 1.1 ‰ is used. In the calculations the mean compressive strength of 27.5 MPa corresponding to the strength of cores with ASR cracks parallel to the load direction is used. The mean compressive strength of concrete cores with cracks perpendicular to the load direction is measured to 16.6 MPa.

The tested shear capacities of the three beams shows results both above (for minor shear span to effective depth ratio) for beam 3.1 and below (for larger shear-span to effective depth ratio) for beams 3.2 and 3.3 the calculated shear capacities where the ASR induced pre-stress in the longitudinal reinforcement is included in the calculations. The ratio between the calculated shear capacity and the tested shear capacity varies from 0.85 to 1.09. According to [21] the original compressive strength of the concrete without inclusion of the ASR cracks is estimated to 42.5 MPa by means of fluorescence microscopy. The calculated shear capacity using the original strength is approximately 0.91MPa. The tested shear capacities show that apparently no significant reduction of the shear capacity had occurred even by severe ASR damages where the concrete compressive strength is reduced by more than 60 % (concrete cores with ASR cracks perpendicular to the load direction) and 35 % (concrete cores with ASR cracks parallel to load direction).

5 CONCLUSION

The main preliminary conclusions obtained from the studies on the influence of ASR on the shear capacity and compressive strength of real-life bridges are:

1. The ASR crack orientation in the bridge decks has a significant influence on the measured concrete compressive strength. The normal drilling orientation in a bridge deck giving concrete cores with ASR cracks perpendicular to the load direction shows a very conservative concrete compressive strength.
2. The modulus of elasticity of the drilled concrete cores has decreased, especially for concrete cores with ASR cracks perpendicular to the load direction. However this may not necessary means that the bending stiffness of ASR damaged reinforced concrete beams has decreased in the same rate.
3. The shear test results indicate that the pre-stress in the reinforcement induced by the ASR expansion can significantly compensate for the loss of compressive strength in real-life bridges.

6 REFERENCES

- [1] Plum, NM (1961): Temporary guide in prevention of damaging ASR in concrete (in Danish). The Danish National Institute of Building Research, Copenhagen, Series SBI
- [2] Wigum, BJ, Pedersen, LT, Grelk, B, and Lindgård, J (2006): State-of-the-art report: Key parameters influencing the alkali aggregate reaction, PARTNER, Report 2.1., SINTEF Building and Infrastructure, Trondheim, Norway
- [3] Nerenst, P (1951): Concrete technological studies in U.S.A. (in Danish), The National Institute of Building Research, Copenhagen, Series SBI Studie No. 7
- [4] Idorn, GM (1967): Durability of Concrete Structures in Denmark, DABI, Holte, Denmark
- [5] Chatterji, S, Thaulow, N, and Jensen, AD (1981): Studies of alkali-silica reaction.4. Effects of different alkali salt solutions on expansion, Cement and Concrete Research, 17, pp 777-783
- [6] Anon (1985): Test Method TI-B51, Alkali Silica Reactivity of Sand, Technological Institute, Copenhagen, Denmark
- [7] Basic Concrete Description (in Danish) (1987), Building Research Institute, Copenhagen, Denmark
- [8] Bach, T, Thorsen, TS, and Nielsen, MP (1993): Load carrying capacity of structural members subjected to alkali-silica reactions, Construction and Building Materials, Volume 7, Number 2, pp 109-115
- [9] The Road Directorate, Denmark (1990): Load Carrying Capacity of structural members subjected to alkali-silica rections, Ministry of Transport, ISBN:87-88728-44-7
- [10] Chana, PS and Korobokis, GA (1991): Structural performance of reinforced concrete affected by alkali-silica reaction: Phase I, British Cement Association, Contractor report 267
- [11] Ahmed, T, Burley, E, and Rigden, S (1998): The static and Fatigue Strength of Reinforced Concrete Beams Affected by Alkali-Silica Reaction, ACI Materials Journal, 95 (4), pp 376-387
- [12] Schmidt, JW, Hansen, SG, Barbosa, RA, and Henriksen, A (2014): Novel shear capacity testing of ASR damaged full scale concrete bridge, Engineering Structures, Volume 79, pp 365-374
- [13] Hansen, SG, Barbosa, RA, Hoang, LC, Hansen, KK (2016): Shear Capacity of ASR Damaged

- Structures – In-depth analysis of some in-situ shear tests on bridge slabs , 15th International Conference on Alkali Aggregate Reaction, São Paulo, Brazil
- [14] Mørup, H, von Scholten, C, and Wegan, V (2013): Have ASR damaged bridges sufficient strength? (in Danish), Traffic and Roads, The Danish Road Directorate
- [15] Danish Standard (2002): DS/EN 12390-3-2009 Testing hardened concrete – Part 3: Compressive strength of test specimens
- [16] The Danish Road Directorate (2010): Guide in loading- and basic calculations (in Danish), Road Regulation Council
- [17] Introduction to Strain Gage Technology, Vishay Measurements Group Educational Program for Strain Gage Technology, Vishay Precision Group
- [18] Rigden, SR, Salam, JM, and Burley, E (1992): The influence of stress intensity and orientation upon the mechanical properties of ASR affected concrete, The 9th International Conference on Alkali-Aggregate Reaction in Concrete, London, UK, pp 865-876
- [19] Danish Standard (2008): Eurocode 2. Design of concrete structures – Part 1-1: General rules and rules for buildings
- [20] Danish Standard (2013): The Danish National Annex for Eurocode 2
- [21] Thaulow, N, Jensen, AD, Chatterji, S, Christensen, P, and Gudmundsson, H (1982): Estimation of the compressive strength of concrete samples by means of fluorescence microscopy, Nordisk Betong, Vol. 26, Nordiska Betonfoerbundet, pp 51-52

TABLE 1: Results of three point bending test and comparison of test results with calculations according to the shear model in Eurocode 2.

Beam no.	a/d ¹⁾	$\tau_{\text{test}}^{2)}$ (MPa)	Compressive strength ³⁾ $f_{c, \text{parallel}}$ (MPa)	Estimated original f_c ⁴⁾ (MPa)	Measured strain ⁵⁾ , ϵ (‰)	$\tau_{\text{Cal}}^{6)}$ (MPa)	$\tau_{\text{test}}/\tau_{\text{Cal}}^{6)}$	$\tau_{\text{cal}}^{7)}$ (MPa)	$\tau_{\text{test}}/\tau_{\text{Cal}}^{7)}$
3.1	3.92	0,94	27.5	42.5	1.1	0,92	1,02	1,1	0,85
3.2	3.24	1,06	27.5	42.5	1.1	0,91	1,16	1,1	0,96
3.3	2.56	1,20	27.5	42.5	1.1	0,91	1,32	1,1	1,09

¹⁾ Shear span to effective depth ratio

²⁾ Tested shear capacity

³⁾ Average compressive strength of concrete cores with ASR cracks parallel to the load direction

⁴⁾ The original compressive strength estimated by fluorescence microscopy

⁵⁾ Average measured ASR induced tensile strain in reinforcement bars

⁶⁾ Calculated shear capacity based on the estimated original compressive strength without inclusion of ASR induced pre-stress

⁷⁾ Calculated shear capacity based on the measured compressive strength of concrete cores with calculated inclusion of ASR induced pre-stress

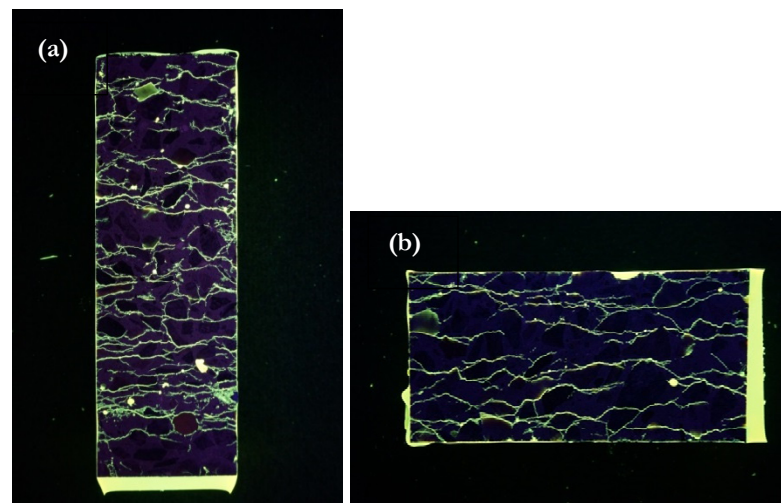


FIGURE 1: Fluorescent impregnated concrete cores drilled vertically (a) and horizontally (b) from bridge no. 3. The crack pattern is typical for all the three severe ASR damaged bridge decks. However the amount of cracks varies between the bridge decks and areas within the same bridge deck. The ASR induced cracks are orientated parallel to the reinforcement bars. The ASR expansion is not restricted in the vertical direction, since the bridge slabs are not provided with vertical reinforcement.



FIGURE 2: Bridge no. 1 - Fine longitudinal wet cracks with white precipitations in the bridge deck and in the prestressed main beam.



FIGURE 3: Bridge no. 3 – Overview from beneath of the pile supported reinforced bridge deck.



FIGURE 4: Bridge no. 3 – Longitudinal and transverse ASR induced cracks with white precipitations from beneath the bridge deck. Locally there is sign of reinforcement corrosion.

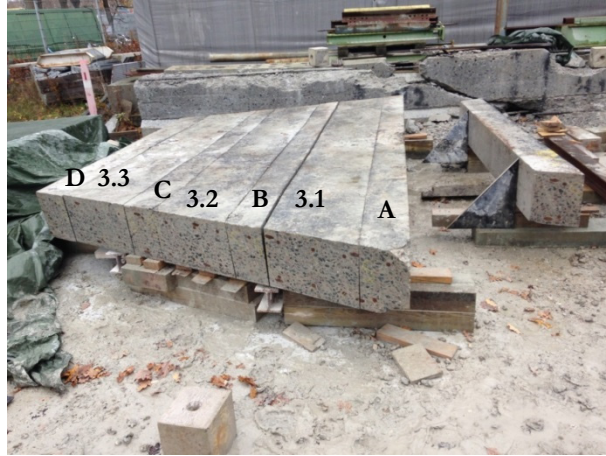


FIGURE 5: Bridge no. 3 - Trapezoidal slabs cut into beams at the Technical University of Denmark. Beam no. 3.1, 3.2 and 3.3 were loaded until failure in a three point bending test setup. Concrete cores were drilled from beam A, B, C and D after strain gage measurements of the ASR induced tensile strain in the reinforcement bars.

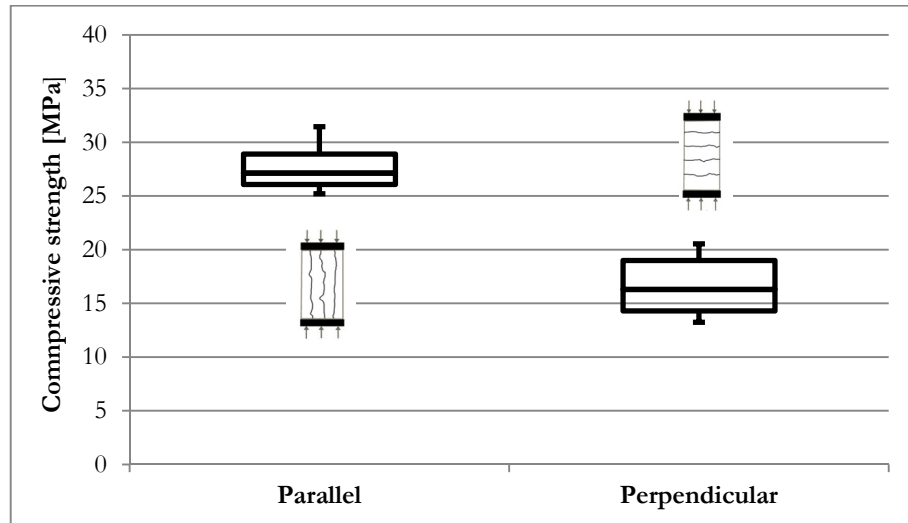


FIGURE 6: Measured compressive strength of concrete cores with cracks parallel and perpendicular to the load direction. The results are presented graphically as a box and whisker plot. The bottom and top of the boxes represents the first and third quartiles and the line inside the boxes represents the second quartile (the median). The top and bottom of the whiskers represents the measured maximum and minimum compressive strengths, respectively.

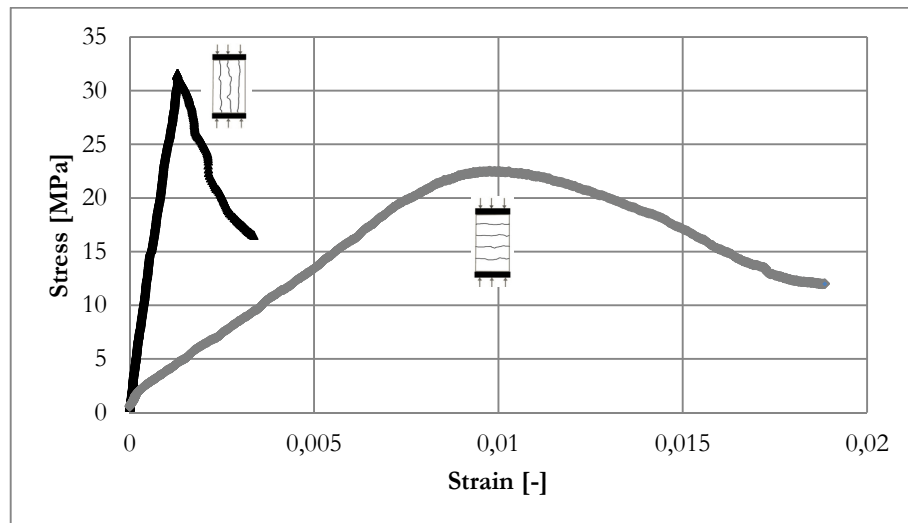


FIGURE 7: Stress-strain relationship for drilled concrete cores with ASR cracks orientated parallel and perpendicular to the load direction. The shapes of the curves are representative for the concrete cores tested from all three examined bridge slabs.

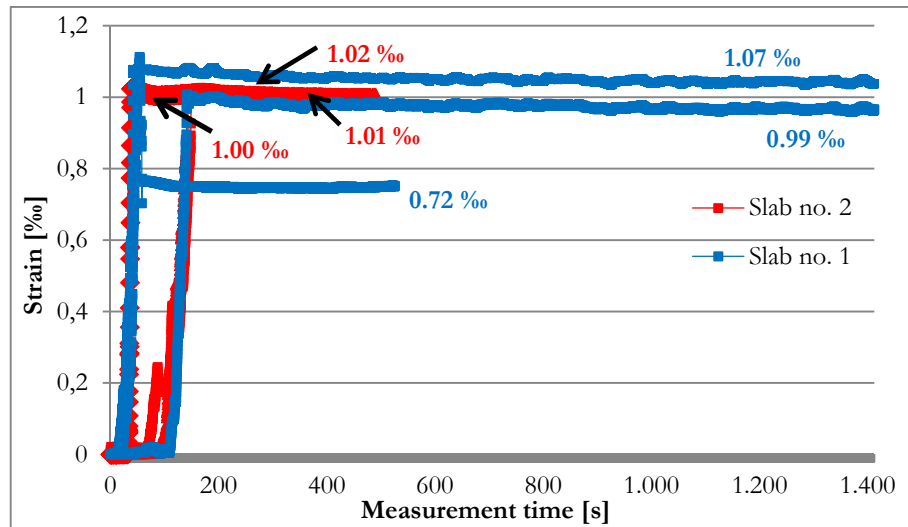


FIGURE 8: Measured ASR induced tensile strain in reinforcement bars as function of time. 3 strain gage measurements are performed on the longitudinal reinforcement bars on beams cut from slab no. 1 and slab no. 2, respectively.



FIGURE 9: Surface condition of concrete beam in slab no. 1 (left) and slab no. 2 (right). Slab no. 1 has considerably more ASR induced cracks than slab no. 2. There are locally signs of surface corrosion on transverse reinforcement bars in slab no. 1, but no sign of corrosion in the longitudinal reinforcement bars where the strain gage measurements are conducted.

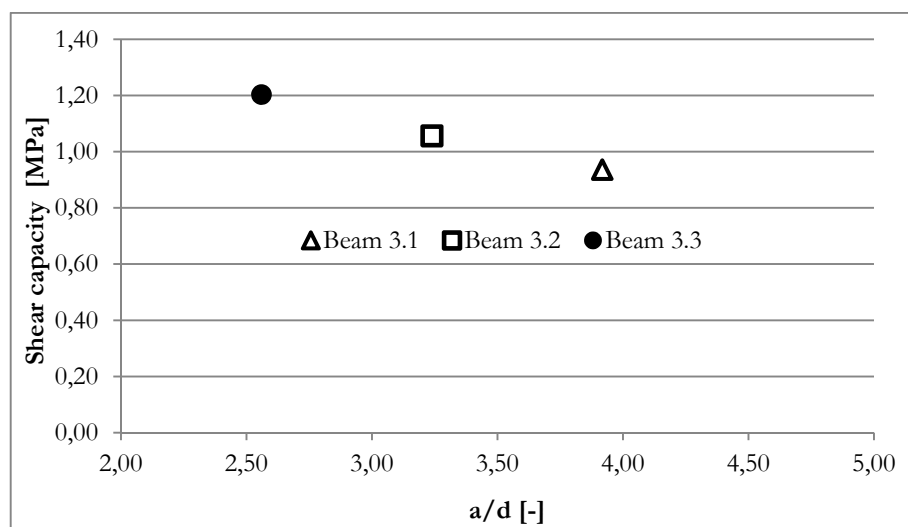


FIGURE 10: Tested shear capacity as function of shear span to effective depth ratio, a/d , from beam no. 3.1, 3.2 and 3.3. The presented beams are cut from the same trapezoidal slab.